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DISCUSSION
OF PROCEEDINGS - SEPARATES

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STRUCTURAL DIVISION

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DISCUSSION OF EAST ST. LOUIS VETERANS MEMORIAL BRIDGE PROCEEDINGS-SEPARATE 150

W. H. JAMESON,[5] M. ASCE.—This discussion presents, from the viewpoint of a member of the engineering staff of the fabricator, some of the problems that were encountered in the long-span cantilever crossing the Mississippi River at East St. Louis, Ill. Also the writer will comment on the modification in (b/t)-ratios for components of compression members as used for this structure.

In the case of Mr. Wyly's formula for plate thickness in compression members, there are two points of deviation from standard specifications such as those of the AASHO and the American Railway Engineering Association (AREA) which deserve further study on the part of specification writers. The first is the question as to whether b should be the full width of the plate, or the distance between the nearest lines of connecting rivets. Mr. Wyly has based his decision to use the full width on the test reports on compression plates for the Delaware River Bridge at Philadelphia, Pa.; the writer believes that there is so much scatter in these test results that either value of b could be used logically, and he recommends that further testing should be carried out, probably in connection with the program of the Column Research Council.

The other question involves the use of a formula that omits all consideration of the allowable unit stress. The allowable unit stress determined by any usual column formula is supposed to be that average unit stress on the section which will keep the maximum unit stress on the extreme fiber within the stress determined by dividing the yield point of the material by the factor of safety. Therefore, in the case of a member with an actual unit stress and an allowable unit stress of the same value, no change in the (b/t)-ratio should be permitted—no matter how small this unit stress may be—because a plate situated at the extreme fiber may be considered already stressed to the maximum. It is only when the actual unit stress is less than the allowable unit stress that a change should be permitted, and the method used in the AREA specifications is the theoretically correct method. In the AREA specifications, the (b/t)-ratio for carbon webs is 32 and for carbon cover plates it is 40, but these ratios may be increased by the ratio $(s_{allow}/s_{actual})^{\frac{1}{2}}$. However, the writer agrees with Mr. Wyly's basic requirement that the same limits for b/t should apply to both the webs and the cover plates.

The discourse in the paper concerning the welded shoes at the main piers is an indication of the close cooperation that existed between the consulting engineers and the engineering staff of the fabricator; and it indicates how such cooperation can result in a better structure, usually with a saving in cost. At the main pier of a large cantilever there is always the possibility of developing severe secondary stresses—caused by the rotation of the span as a whole as the live loading moves from main span to anchor arm and by the deformations of the members themselves. After considerable discussion between the consulting engineers and the fabricator's engineers it was decided that it was better to provide for span rotation by using an articulated shoe, as

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shown in Fig. 2, than to attempt to reduce secondary stresses by using pin-ended members at a shoe that could not rotate. Mr. Wyly's measurements, on which the author has provided a preliminary report, but which were not available when this discussion was prepared, should prove whether this procedure has tended to minimize the secondary stresses, and whether rotation of the span as a whole with respect to the pier actually has occurred during erection and concreting and subsequent to completion of the bridge.

Another example of the close cooperation between the consulting engineers and the fabricator's engineers is in the design of the main truss members. For complex reasons, it had been necessary to design these members in a very short time, and without complete study as to how they could be fitted together satisfactorily. After the award of the contract, the members were re-designed to the satisfaction of all parties concerned, and a better structure, with consistent details throughout, was the result. Attention is called to the use of 5-in. by 5-in. angles as the corner angles for nearly all box members. When 4-in. by 4-in. angles, or 6-in. by 6-in. angles with double gages are used, there is always difficulty at chord splices in getting rivets through outstanding 4-in. legs, or through outstanding 6-in. legs on the inner gage line, because of the packing out of splice plates over the web legs of the angles. The use of 5-in. by 5-in. angles, with single gages in each leg, greatly facilitates field riveting and consequently results in better rivets. Some concern was expressed that there would be a tendency for the heel of these 5-in. by 5-in. angles with rivets on a single gage line to pull away from the edge of the web or cover plates. Close inspection of the members revealed no such difficulty, and no special care had to be exercised by the fabricating shop to guard against this.

The design of the floor steel presented some difficulties to the erector, and the solution described here was greatly facilitated by the close cooperation of the consulting engineers. The stringers framed to the floor beam web, in some cases between floor beam stiffeners, and the outstanding legs of these stiffeners made it impossible to erect the stringers by swinging them in. One framing angle for the stringer to the floor beam connection was shop riveted to the floor beam web, and the other was loose. It was decided, therefore, to eliminate vertical stiffeners on one side of the floor beam web, and to use a single longitudinal stiffener on that otherwise plain side. This longitudinal stiffener was located below the stringers, and thus the stringers could be swung in freely without interference at that end away from the outstanding legs of the vertical stiffeners.

The main structure, as now standing, is a prime example of teamwork between designer and fabricator. The result proves that such cooperation, which modifies the construction to utilize most effectively the available fabricating and erecting facilities, pays real dividends in a better structure and, usually, in a financial saving.

JOSEPH SORKIN,[6] M. ASCE.—Among the topics in this paper is a discussion of the modification of AASHO specifications as regards the design of main compression members. These specifications, as well as other standard design

[6] Partner, Howard, Needles, Tammen & Bergendoff, Cons. Engrs., Kansas City, Mo.

specifications, are intended for relatively short spans. In the design of longer spans it behooves one to explore the fundamental concepts regarding the stability of thin plates in compression. The importance of the structure under consideration fully justifies the more thorough and accurate approach to the problem from the standpoint of safety as well as economy. The designers of the bridge are to be commended for that reason.

For relatively heavy reactions of a long span of the magnitude of the Veterans Memorial Bridge, a line bearing is definitely preferable to the conventional shoes in which pins transfer the loads to the supports. Forces

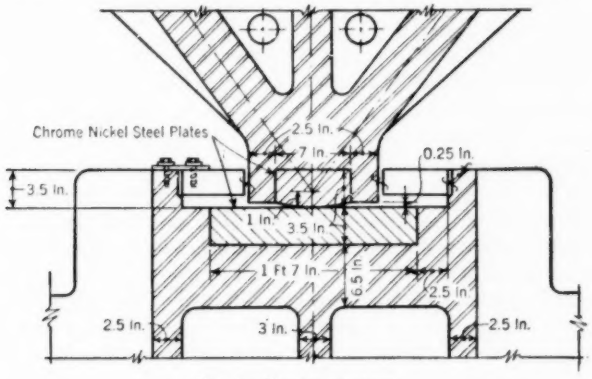

FIG. 3.—Bearing Surfaces.

engendered by pin-type shoes, in the frictional resistance to articulation, may cause significant secondary stresses in the truss members in the immediate vicinity of the bearing, as well as adversely affecting the supporting piers. The type of shoe used for the Veterans Memorial Bridge has been used in several long-span bridges. Instead of special alloys for the bearing slabs, it has been found advantageous to provide inserts of chrome nickel alloys into slabs made of ordinary cast steel. The inserts, made of alloy with a high elastic modulus, may be subjected to considerably higher bearing concentrations. Fig. 3 shows the details of this arrangement. The chrome nickel plates are press fitted into the shoes, the bearing surface of the upper plate having a radius of 8 ft. Incidentally, the same type of bearing may also be used for expansion shoes by setting the lower casting on alloy rollers.

Vertical dimensions of the truss framework at the ends of the 470-ft end spans are not given. As shown in Fig. 1, it appears that the inclination of the end posts is at an unusually acute angle. In certain instances it has been that with end posts sharply inclined there is a tendency toward considerable vibration of the span under live load. This condition apparently results from the fact that the framework in the immediate vicinity is subjected to bending moments analogous to a rigid frame, thus resulting in correspondingly greater deflections.

Details of structures are important and their description is of interest. However, the writer is of the opinion that a recorded history of the principal

aspects of planning and construction of projects of this magnitude is of even greater importance and interest. Thus, the value of the paper could be greatly enhanced if, in addition to the details described by the author, information were furnished as regards other basic problems related to the project. It is hoped that in the closing discussion the author will present the following data:

1. Conditions dictating the lengths of spans;
2. Other types of structures considered and the reasons for adopting cantilever trusses;
3. Basis for establishing the length of the suspended span and of the cantilever arms as used;
4. A description of erection methods employed for the superstructure;
5. Any other controlling data relative to the design and construction of the bridge; and
6. A general breakdown of cost including unit prices for principal construction items.

A. L. R. SANDERS,[7] M. ASCE.—The discussion that this paper has received and the additional design and construction problems to which the discussers draw attention are most gratifying.

Mr. Jameson is correct in his recommendation that further tests be carried out with respect to the question as to whether b, in the expression for the (b/t)-ratios for components of compression members, should be the full width of the plate or the distance between the nearest lines of connecting rivets.

The type of shoe shown in Fig. 3 is an interesting variation from that shown in the paper. The writer believes that Mr. Sorkin's design would show greater economy as the load to be carried increases, when compared with that used at East St. Louis. Mr. Sorkin's remarks with respect to the inclination of the end post drew attention to an error in the proportions of Fig. 1. The end post has a rise of 45 ft in a distance of 50 ft, and the depth of the center span is 50 ft.

For complex reasons, as stated by Mr. Jameson, the selection of the type of structure, the determination of the span lengths, the relative length of the suspended span and the cantilever arms, and the design of the main members had to be accomplished in a short period of time. Studies made by the designing engineers indicated that a cantilever structure having the ratios (anchor span to center span) finally adopted would be the most economical.

The length of the center span and the location of Piers No. 9 and 10 were conditions established by the United States War Department in the issuance of the permit for the construction of the bridge. The span length and pier location were affected materially by the Eads Bridge, which spans the Mississippi River approximately 800 ft downstream from the site of the East St. Louis Veterans Memorial Bridge. The project was constructed with a lump sum type of contract, and unit prices for the various construction items are not available.

[7] Partner and Chf. Engr., Hazelet & Erdal Cons. Engrs., Chicago, Ill.

DISCUSSION OF RAPID COMPUTATION OF FLEXURAL CONSTANTS
PROCEEDINGS-SEPARATE NO. 170

HENRY MALTER,[2] A. M. ASCE.—A method of numerical integration that

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differs from the author's method for computing stiffnesses, carry-over factors, and fixed-end-moment factors has been developed by N. M. Newmark,[3] M. ASCE. Mr. Newmark's method involves no new theoretical concepts; how-

[3] "Numerical Procedure for Computing Deflections, Moments, and Buckling Loads," by N. M. Newmark, *Transactions*, ASCE, Vol. 108, 1943, pp. 1161–1234.

ever, by a systematization of computations an answer can be obtained quickly. As in any numerical procedure, exact solutions are not found, but the degree of accuracy obtained is sufficient on consideration of all the variables involved in the problem that is ordinarily encountered. The accuracy of any solution can be improved by increasing the number of subdivisions of the beam. After becoming familiar with the technique of solution, the matter of subdivisions is one of individual discretion. The time spent must be balanced against the accuracy desired, with a constant realization of the limitations on accuracy imposed by the conditions of the problem.

General Solution.—Using the method developed by Mr. Newmark, a moment is introduced into beam AB at end B in Fig. 5(*a*). The moment diagram is changed into a series of concentrated angle changes α that are considered to be loads in Fig. 5(*b*). With zero slope at A, and progressively adding each concentrated angle change α, the final deflection at B and the shape of the elastic

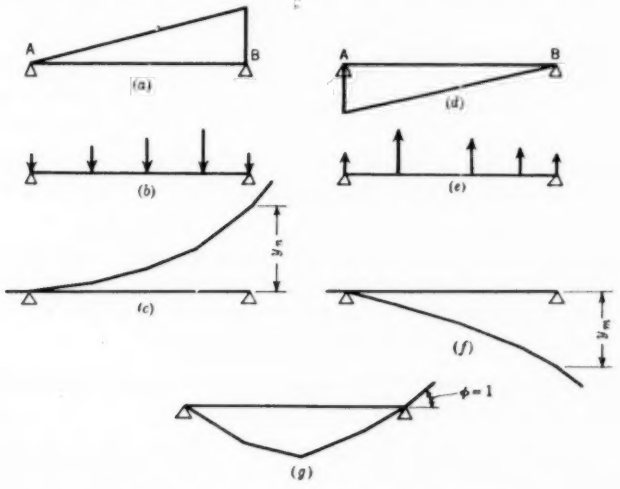

FIG. 5.—BASIC CONCEPTS OF THE NEWMARK METHOD

curve are obtained as shown in Fig. 5(*c*). To correct for the deflection y_n at B, a moment is introduced at A in the correction diagram of Fig. 5(*d*). This results in the series of load concentrations shown in Fig. 5(*e*). With zero slope at A, the concentrated angle changes are added, thus obtaining the deflection y_m at B (Fig. 5(*f*)). If this deflection of B is of opposite sign to that obtained in Fig. 5(*c*), y_n/y_m times the correction diagram (Fig. 5(*f*)) is added to the deflection diagram (Fig. 5(*c*)) to form a deflection diagram having zero slope at A and zero deflection at B (Fig. 5(*g*)). The slope at B is reduced to unity and the moments and the deflections are reduced accordingly. This produces a unit rotation at B, and the moment accompanying this rotation is the beam stiffness. The deflected line is an influence line for the moment at B, using the Müller-Breslau principle, and the carry-over factor is the ratio of the final corrected moment at A to the moment at B.

Sign Convention.—A numerical procedure requires a sign convention for rapid computations, and the convention adopted by Mr. Newmark will be retained. Any method of obtaining panel concentrations from distributed loads is approximate. A suggested procedure is illustrated in Fig. 6.

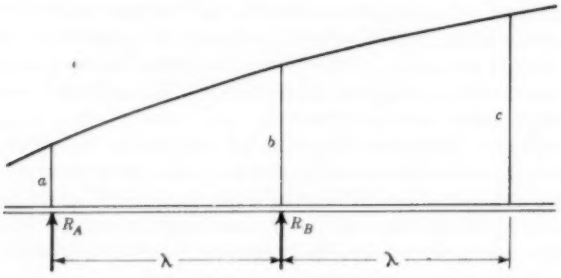

Fig. 6.—Loading Curve

The formulas derived by Mr. Newmark,

$$R_A = \frac{\lambda}{12}\frac{1}{2}(7\,a + 6\,b - c) \quad \ldots\ldots\ldots\ldots\ldots\ldots (15a)$$

and

$$R_B = \frac{\lambda}{12}(a + 10\,b + c) \quad \ldots\ldots\ldots\ldots\ldots\ldots (15b)$$

assume that the loading curves are parabolic and continuous. For problems involving discontinuities, two R_A-values are combined.

Numerical Integration.—The steps to be used are illustrated in Table 3 and are as follows (the lines refer to the lines in Table 3):

a. A moment M_b (some multiple of the number of subdivisions) is applied to the beam AB at point B (line 6).

b. The moment values at the subdivisions are changed to α-values $\left(\text{equal to } \frac{M}{E\,I}\right)$ at the particular point (line 7).

c. The distributed α-values are changed to $\bar{\alpha}$-concentrations by using the appropriate relationship. The factor $h/12$ is included in the right-hand column (line 8).

d. Assuming zero slope at point A, the slope in each subdivision is computed by adding each $\bar{\alpha}$-concentration successively (line 9).

e. Assuming zero deflection at point A, the deflection at each point is determined by successively adding the various slopes (line 10). The factor h is included as a common multiplier in changing the factor to $h^2/12$.

This satisfies the condition of the problem that the slope at point A be zero. However, the second condition—that the deflection at point B be zero—is not satisfied. To make the necessary correction, a correction-configuration diagram must be added by applying a moment at point A.

f. A moment M_a is applied at end A (line 1).

g. The moment values are changed to α-values (line 2).

h. The $\bar{\alpha}$-concentrations are computed (line 3).

i. Assuming zero slope at point A, the slope in each subdivision is computed (line 4).

j. Assuming zero deflection at point A, the deflection at each point is successively determined (line 5).

k. To make the deflection at point B zero, it is necessary to add a proportion of the deflection found in line 5 to that found in line 10. The correction ratio is the deflection y_m at point B (line 10) divided by the deflection y_n at point B (line 5) with the appropriate sign to produce a total deflection at point B equal to zero. This correction ratio is then multiplied by each of the values in line 5 to obtain y_c (line 11).

l. Lines 10 and 11 are then added to determine the final deflections (line 12).

m. The slope ϕ at point B is computed. This consists of three steps: The first step is the determination of the slope to the left of point B, the second step is the computation of $\bar{\alpha}$ at point B, and the third step is to determine the corrected value of $\bar{\alpha}$ from the correction configuration.

These steps allow for the determination of the stiffness, the carry-over factor, and the fixed-end-moment factor for both distributed and concentrated loads for point B.

To illustrate the application of Mr. Newmark's method, the author's first problem is analyzed in Table 3. Inasmuch as the stiffness is infinite at both

TABLE 3.—SOLUTION OF ILLUSTRATIVE PROBLEM

Line	Term or factor	Distance from Left Support, in Feet				
		0.00	3.25	6.50	9.75	13.00
1	Applied moment, M_a	0	+6	+12	+18	+24
2	α-values	Determined semi-graphically				
3	$\bar{\alpha}$-concentrations	−1.28	−17.97	−39.00	−30.39	−0.17
4	Slope, ϕ	+87.53	+69.56	+30.56	+0.17	
5	Deflection, y_n	−187.82	−100.29	−30.73	−0.17	0
6	Applied moment, M_b	+24	+18	+12	+6	0
7	α-values	Determined semi-graphically				
8	$\bar{\alpha}$-concentrations	−4.64	−45.10	−39.00	−14.38	−0.05
9	Slope, ϕ	+98.53	+53.43	+14.43	+0.05	
10	Deflection, y_m	−166.44	−67.91	−14.48	−0.05	0
11	Deflection, y_c	+166.44	+88.71	+27.30	+0.15	0
12	Deflection, y	0	+20.80	+12.82	+0.10	0

ends for varying distances, resulting in zero $\left(\frac{M}{E\,I}\right)$-area, a semigraphical method was used to obtain the α-values at the evenly-spaced points.

For this problem, the slope ϕ at point B was

$$\phi = +20.80 + 4.64 - 1.13 = 24.31 \ldots\ldots\ldots\ldots (16a)$$

the stiffness was

$$K_a = \frac{24 \times 4 \times 3.25}{24.31} \frac{E\,I}{L} = 30.0 \text{ kip-ft} \ldots\ldots\ldots\ldots (16b)$$

the carry-over factor was

$$C_{ab} = \frac{166.44}{187.82} = 0.887 \ldots\ldots\ldots\ldots\ldots\ldots (16c)$$

and the fixed-end-moment factor was

$$F_{a,\,0.5} = \frac{12.82\,P\,h}{24.31} = 0.132 \ldots\ldots\ldots\ldots\ldots (16d)$$

In a similar manner, the stiffness, the carry-over factor, and the fixed-end-moment factor for the right-hand side of the bent were computed. To compare the results obtained by the use of Mr. Newmark's method with those obtained by the author, Table 4 has been prepared. This table is presented to demonstrate the technique involved, and not to question the physical meaning of the constants derived. Although beam dimensions may take on sudden changes, distributed angle changes (moment areas) are stress-strain relationships, and all changes must be gradual.

TABLE 4.—COMPARISON OF RESULTS

Method	Term or Factor					
	K_a, in kip-feet	C_{ab}	$F_{a,\,0.5}$	K_b, in kip-feet	C_{ba}	$F_{b,\,0.5}$
Morrison	30.4	0.894	0.143	39.6	0.680	0.205
Newmark	30.0	0.887	0.132	38.6	0.680	0.216

PAUL ROGERS,[4] M. ASCE.—Although numerical methods such as moment distribution and relaxation system have become so widely accepted that the classical analyses for most computations are practically obviated. Their use is usually limited to prismatic members. There are tabulated constants for straight and parabolic haunches available, but these consider only a few of the most commonly applied loadings.

[4] Cons. Engr., Rogers and Snitoff, Inc., Cons. Engr., Chicago, Ill.

The fundamental theory of moment distribution supplies the tools with which the stiffness factors, distribution factors, carry-over factors, and the proper fixed-end moments can be computed for almost all possible loadings and shapes of the supporting members. In actual practice, however—unless

the structure is of utmost importance, or is repeated to such an extent that even minute savings can amount to considerable economies—designing engineers cannot afford the time necessary to make refined computations.

Mr. Morrison recognized that the tedious computations for the required constants are made laborious by needless repetitions of numerical computations. By proper grouping, registering, and tabulating the obtained values, the author's method allows for their use in further computations, at once saving time and eliminating some sources for numerical errors.

The only systems to be considered in practical design (apart from academic studies or research), are those which afford speedy and accurate results. It has been found that even for prismatic members there is a need for tabulated values for fixed-end moments because loadings on structures are complex and usually unsymmetrical. The writer has tabulated fixed-end moment constants.[5] Because of the unlimited variety of possible shapes for the supporting members, the constants are limited to prismatic members.

[5] "Tables and Formulas for Fixed End Moments of Members of Constant Moment of Inertia," by Paul Rogers, Frederick Ungar Pub. Co., New York, N. Y., 1953.

Mr. Morrison's basic formulas may be varied to such an extent that his numerical computations might be combined with tabulated constants. Such combination would give the moment-distribution method widespread practical application.

THOMAS G. MORRISON,[6] A.M. ASCE.—Mr. Malter has reviewed Mr. Newmark's approximate method for obtaining deflection curves and has applied the method to the first illustrative problem. It is of interest to note that, for the triangular loading used by Mr. Malter, the slopes ϕ (given in lines 4 and 9 of Table 3) are, respectively, $\phi_{1,r} \frac{I_0}{I} M L$ and $(\phi_{0,r} - \phi_{1,r}) \frac{I_0}{I} M L$ in which $M L$ is the product of the applied moment (24 ft kips) and the beam length (13 ft). These values can be obtained more quickly and accurately by use of the flexural functions in lieu of the semigraphical integration used by Mr. Malter. For a less simple problem—for example the second illustrative problem—the flexural functions would certainly give the slopes more accurately and quickly. Similarly, for the loadings used, the exact values for the deflections y_n are proportional to $(r \phi_{1,r} + \phi_{2,r})$ and the deflections y_m are proportional to $(r' \phi_{1,r} - \phi_{2,r})$.

[6] Cons. Engr., Chicago, Ill.

Several other well known methods have been used for obtaining the flexural constants—the column analogy and the conjugate beam are two of the more popular. Most of these methods are simple in physical theory but entail more numerical work than is necessary when used for computing flexural constants. Because the flexural functions are the "building blocks" of which the beam constants are built, all of these methods can be expressed in terms of the flexural functions when arranged for the computation of the flexural constants.

Mr. Rogers refers to the many excellent tables that are available for obtaining fixed-end moments. These tables cite many of the conditions encountered in building construction. However, the data published for continuous bridge design are incomplete, and (for the reason given by Mr. Rogers) must so remain. Lack of a suitable, quick, accurate method for obtaining the flexural constants has influenced bridge designers to adopt soffit curves solely because

tabulated flexural constants were available for the curves used—to the economic and artistic detriment of the design. The ungainly, half-horizontal, half-parabolic soffit curve used for the end spans of so many otherwise beautifully-proportioned concrete bridges is a case in point. Introduction of the flexural functions and the tabulated values for $\phi'_{0,r}$, $\phi'_{1,r}$, and $\phi'_{2,r}$ has freed the bridge engineer and architect from the few tabulated forms for soffit curves. Bridges can be proportioned for economy and grace without the necessity for an analytic expression and without increasing the difficulty of design.

Corrections for Transactions.—On page 3, line 3, change "* * * varying as x^{m-2}" to "* * * varying as x^{m-3}." On pages 4 and 5, in Eqs. 12*a* and 12*b*, change the lower limits on the summation signs from "$q = r_1$" to "$q = r_0$," and change the upper limit on the summation signs from "$q = r_n$" to "$q = r_{n-1}$." On page 5, in Eq. 12*c*, change the upper limit on the summation sign from "$q = r_n$" to "$q = r_{n-1}$." On page 8, in the second paragraph from the bottom of the page, change "$F'_{1,0.5}$" to "$F'_{b,0.5}$" and change "$F_{0,0.5}$" to "$F_{a,0.5}$;" and on page 8, in the sixth line from the bottom of the page, change "* * * yield $F_{1,0.5}$" to "* * * yield $F_{b,0.5}$."

DISCUSSION OF THE AMPLIFICATION OF STRESS IN FLEXIBLE STEEL ARCHES PROCEEDINGS-SEPARATE NO. 245

Milo S. Ketchum,[1] A. M. ASCE.—The writer is particularly interested in this paper because the same problem was discussed in connection with the description of the design of the steel arches for the University of Wyoming Fieldhouse.[2] In this article all stress values are given and it will be of interest to compare the results by the two methods.

In the writer's analysis the amplification of stress is:

$$(411 - 380)/380 = 8.2 \text{ per cent}$$

In the author's paper the following constants are required:

$$L = 110.5 \text{ ft.}, \quad r \text{ (for 36WR150)} = 14.29$$

$$L/r = 110.5 \times 12/14.29 = 92.7$$

The ratio of axial to bending stress is $3.62/9.80 = 0.370$.

The reduced L/r value using the coefficient 0.90 for a two hinged arch is $92.7 \times 0.90 = 83.3$.

Using the above values in Fig. 2 of the author's paper the amplification ratio is about 10 per cent. The writer considers this to be in very good agreement with the more precise method.

This paper should be very helpful in making a preliminary estimate of the amplification or buckling stresses in steel arches, particularly for those used in buildings for which there is not the time available to make long and elaborate studies.

1. Consulting Engineer, Denver, Colorado.
2. "Steel Arch Analyzed and Designed by Semigraphical Methods" by Milo S. Ketchum, Civil Engineering Aug. 1952. pp 30-34.

DISCUSSION OF AN ENGINEERING APPROACH TO BLAST RESISTANT DESIGN
PROCEEDINGS-SEPARATE NO. 306

Dr. S. J. Fraenkel.—The writer welcomes the advent of this paper to the slowly but steadily increasing literature on structural dynamics. Aside from the appeal which this subject holds as a technical problem, recent newspaper headlines and stories are lending ample emphasis to the practical and immediate implications of this subject. In the writer's opinion, this contribution to the as yet sparse unclassified literature in this field has the specific virtue of presenting the problem in terms of design rather than of analysis. Although many procedures previously advocated, including those proposed by the writer, can be used on a trial and error basis for design purposes, they are not as conveniently or efficiently applicable to the design problem as the method proposed in the paper under discussion.

Let us examine, for a moment, the basic approach taken in this paper and compare it with earlier analysis procedures to determine what has made this transformation to the design approach possible. Expressed in its barest mathematical essentials, the problem of both analysis and design revolves around the solution of the following equation for a suitably simplified or idealized structure:

$$m\ddot{x} + R(x)x = F(t)$$

In the so-called analysis procedures, the value of the deflection x is sought as a function of time. The mass m, the building resistance R, and the external force F, which moves the building, are all known at the beginning of the problem. The method thus reduces to the solution of the foregoing differential equation. This solution may be more or less difficult to obtain depending upon the complexity of R and F functions. This paper departs from this approach by recognizing the obvious fact that in the design of new structures the building resistance R is precisely the thing to be determined. Thus R becomes the dependent variable, but, unfortunately, no explicit solution for R can be obtained from this differential equation for arbitrary forcing functions F. In order to overcome this difficulty the author simplifies F so that a closed form solution of the differential equation can be obtained. Moreover, a specific value of the deflection x is selected which the resistance R will not permit to be exceeded. In this manner, through the selection of a fixed maximum deflection x and of a simple analytical forcing function F, the problem can be inverted from its usual formulation and the desired building resistance R can be solved for directly. In cases where the actual forcing function F is too irregular to permit simple idealization, a crude approximation by means of methods outlined in the paper will at least reduce the number of trials required by a more rigorous procedure. There is no question that the simplified approach advocated by the author should be of great help in many design activities.

The writer would like to comment next on the author's omission of the negative phase as a possible agent of destruction. The writer agrees with the legitimacy of this statement and wishes to offer two reasons in support of it

which were not contained in the original paper. First, it is well known that the pressure and force associated with the negative phase is much smaller than that associated with the positive phase and is, say, of the order of one-fifth of the latter. Damage to any structural system requires a certain threshold intensity of loading irrespective of duration, and calculations would undoubtedly show in most instances the pressure level during the negative phase would be below that required to do damage. Second, it should be kept in mind that the direction and distribution of loading during the negative phase will be 180^{o} opposite to that encountered during the positive phase. The resistance of buildings is the same, however, for two diametrically opposite directions, and it is difficult to conceive therefore that a building not already damaged by the higher pressures in the positive phase will be damaged during the negative phase. If the forces exerted during the negative phase were not directly in line with those in the positive phase, one would be a good deal less certain about the advisability of ignoring the effects of the negative phase.

The author's paper, in common with other publications of the writer and of the writer's office, takes account of the vertical loads by adding them to the applied overturning moment. In this particular paper this effect is actually expressed as a reduction of the slope of the resistance displacement function. However, recognition of this effect in these two manners is entirely identical. The writer would like to point out that there exists still another effect of the vertical load which is of potentially very significant importance in problems of this kind but whose complexity has led virtually everyone to ignore it thus far. This effect concerns the changes in the resistance properties of the cross-sections of the columns which is brought about by the simultaneous presence of direct forces and bending moments. Since both the forces and the moments are independent functions of time, enormous complexities would be introduced into procedures of this kind by consideration of this variation. There exists some evidence, however, that this effect may be fully as important as the inclusion of the vertical loads in the driving force and future studies in this field should not ignore this effect without first providing adequate justification.

The author classifies buildings with fifty per cent more openings in the front and rear walls as drag type buildings. The paper does not say anything about possible restrictions which may have to be imposed on the dimensions of interior members lying in the path of a wave traveling through such a drag type building. A number of calculations have been carried out in the writer's office as part of the preparation of a revised edition of a manual on "A Simple Method of Evaluating Blast Effects Upon Buildings" in which quantitative limits of error are stated which apply to the response predictions when interior framework of certain dimensions is present. To put it another way, these calculations state a so-called "average" framework dimension commensurate with a certain acceptable maximum percentage error in the response prediction for a drag building of certain dimensions if this building is still to be considered a drag type building. This limitation must be superimposed on the minimum fifty percent wall opening requirement stated in this paper. In most cases this limitation would not be restrictive from a practical point of view provided one is not too punctilious about the expected accuracy.